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Mr. Valmichael Leos
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1445 Ross Avenue Suite 1200
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Re: San Jacinto River Waste Pits Superfund Site Time Critical Removal Action
Response to USEPA Questions on TCRA Cap Assessment
CERCLA Docket No. 06-12-10

Project Number: 090557-01

Dear Mr. Leos:

On behalf of International Paper Company and McGinnes Industrial Maintenance Corporation (the Respondents), this letter provide responses to USEPA questions on the Time Critical Removal Action (TCRA) Assessment for the San Jacinto River Waste Pits Superfund Site (the Site), which were transmitted via email to Anchor QEA, LLC (Anchor QEA) on April 25, 2013, and received by certified mail on May 6, 2013.

Below are the USEPA questions, with responses provided following each question.

Question:

1. **How was Maynord's equation for stable armor size parameterized? What are the values used for**
    - a. **Safety factor**
    - b. **Stability coefficient**
    - c. **Velocity distribution coefficient**

d. **Blanket thickness coefficient**
e. **Gradation uniformity coefficient**
f. **Depth used for the berm slope and crest (depth of grid cell containing the berm, was it averaged over the 15 meters? Was it assigned to the minimum depth?)**

Response:

As described in Section 5 of Appendix I of the Time Critical Removal Action (TCRA) Removal Action Work Plan [RAWP, Anchor QEA (2010)], predicted current velocities within the TCRA Site were used to calculate the median particle diameter (D_{50}) for the cover material using the Maynord (1998) method. The method presented in Maynord (1998) is based on the U.S. Army Corps of Engineers (USACE) "Hydraulic Design of Flood Control Channels" (USACE 1994). This method uses velocity and flow depth computed by the depth-averaged hydrodynamic model to determine the size of the granular cover material that will be stable for a given current velocity. The following values were used for the coefficients in the Maynord Equation (which is based on USACE 1994):

- Safety factor (S_f) = 1.3 (from page A-6 of Maynord 1998). Per Maynord (1998), the minimum safety factory for rip rap design is 1.1. Although the TCRA was intended as a short-term remedy, a higher safety factor of 1.3 was used for the TCRA to be more conservative and protective.
- Stability coefficient (C_s) = 0.3 for angular rock (from page A-6 of Maynord 1998).
- Vertical velocity distribution coefficient (C_v) = 1.0 (from page A-6 of Maynord 1998).
- Blanket thickness coefficient (C_t) = 1.0 for flood flows and a thickness = D_{100} (from page A-6 of Maynord 1998).
- Gradation uniformity coefficient (D_{85}/D_{15}) = 3.5 for a well-graded material (page A-6 of Maynord 1998).
- The Environmental Fluid Dynamics Code (EFDC) hydrodynamic model grid cells that contained the western berm was based on the maximum elevation that the model grid cell covered. Therefore, the depth in the grid cells that covered the western berm slope and crest represented the western berm crest (i.e., the minimum water depth for that cell, not the average depth).

Question:

2. **What is the measured or estimated grain size distribution for the B/C armor material? Specifically, what are the**
   a. **D_{100}**
   b. **D_{85}**
   c. **D_{60}**
   d. **D_{50}**
   e. **D_{15}**
   f. **D_{10}**
   g. **D_{30}**

Response:

Using the contractor gradation submittal for the B/C armor material, the following is the measured and estimated grain size distribution for this material:

- D_{100} 12 inches
- D_{85} 9 inches
- D_{60} 8 inches
- D_{50} 6 inches
- D_{30} 4 inches
- D_{15} 0.12 inches
- D_{10} 0.033 inches

A grain size distribution curve for this material is attached for reference.

Question:

3. **What was the maximum design slope for the foundation of the West Berm armor?**

Response:

As described in Section 2.2.2 of Anchor QEA (2013), the steepest foundation design slope used in the TCRA Removal Action Work Plan was 2 Horizontal (H): 1 Vertical (V). During the TCRA cap reassessment (Anchor QEA 2013), a western berm foundation design slope of 1H:1V was evaluated.

Question:

4. **How was armor stability evaluated for waves and overtopping? What is the maximum wave height or characteristic wave height?**

Response:

As described in Section 2.1 of Anchor QEA (2013), vessel-and wind-generated waves were calculated for the TCRA Site. Due to the amount of turbulence generated by breaking waves in the surf zone, the armor layer was modeled in the TCRA design as a rubble mound berm; that is, a sloped berm (or revetment) consisting of rock. Armor stone for sloped berms was sized using guidance from USACE (USACE 2006) as part of the original TCRA design. The USACE guidance was used because the methodology to evaluate armor stone sizes for sediment caps presented in USEPA's design guidance (Maynord 1998) does not consider the effects of waves breaking on a cap, as would be the case for the sloped berms at the TCRA Site. The surf zone is defined as the region extending from the location where the waves begin to break to the limit of wave run-up on the shoreline slope. Within the surf zone, wave-breaking is the dominant hydrodynamic process (USACE 2006).

As described in Anchor QEA (2010), wind-generated waves and vessel wakes were expected to be less than 2 feet at the TCRA Site. Specifically, wind-generated waves were estimated to be less than 1.7 feet, and vessel generated wakes were expected to be less than 1.2 feet at the TCRA Site.

Details of vessel and wind-generated wave analysis are included in Section 2.1 of Anchor QEA (2013).

Questions 5 and 6

Because these two questions pertain to the same general subject of combined wave generated and orbital forces, they are presented here together and a unified response is provided.

5. **The 2-D EFDC model runs with vertically averaged velocities will underestimate local shear stress in areas with these steeper slopes because the speeds are greater due to the vertical component. How does the design approach account for the higher vertical velocities and turbulence along face of the slope than modeled in EFDC due to limitations in the grid resolution to represent the actual slope or account for vertical velocities? The model represents the maximum slope as approximately 1V:10H while the actual slope is 1V:2H or greater.**
6. **The reassessment of the west berm analyzed the stability of the armor layer for wave runup and overtopping using techniques from the USACE Coastal Engineering Manual, but did not analyze the stability for sustained flow up and over the west berm. Bottom shear stresses from sustained flow were estimate from the EFDC model runs. The 2-D EFDC model runs with vertical averaged velocities does not include wave effects, which can be sizable for shallow water as along the crest and upper portion of the berm. When the western cell is inundated under extreme flow events such as the 25-yr and 100-yr events and high flow velocities are predicted to occur along and over the west berm, how are the bottom shear stress computed to incorporate the shear stress induced by orbital velocities from waves? Or how does the design approach account for the higher vertical velocities and turbulence along [the] face of the slope induced by waves?**

Response:

The armor stone at berm faces that have the steepest slopes is sized to resist breaking waves. The design is therefore conservative because the required rock size to resist breaking wave forces is higher than the required rock size to resist the combined orbital velocity + current forces. The Safety Factor (S_f) was increased to 1.3 in Maynord's Equation from the recommended 1.1 as a conservative method to account for variations in bathymetry and topography and the associated potential variations in velocities and turbulence intensity for small-scale site variations that are smaller than the two-dimensional EFDC model grid resolution.

Discussion

Outside of the surf zone, orbital velocities from waves combined with currents can increase bottom shear stresses. Combining extreme river current with extreme orbital velocity forces is considered to be very conservative because the probability of both extreme events occurring simultaneously is very low. Nevertheless, in response to USEPA's questions, the following discussion was developed to present additional evaluations for such conditions.

As described in Section 2.1 of Anchor QEA (2013), the armor stone is designed to resist forces due to waves breaking on the TCRA cap (that is, waves would propagate and break on the western berm armor stone). Within the surf zone (the location where waves break), wave-breaking is the dominant hydrodynamic process (USACE 2006).

An example is provided below to demonstrate how designing the armor stone to resist breaking waves will also protect against combination of bottom velocities due to superimposed wave and current forces when the berm is overtopped. Two methods were used as a comparison: 1) calculation of the combined bottom shear stresses due to waves, and 2) currents and the use of an orbital velocity-based equation presented in Maynord (1998).

Method 1 – Combined Current/Wave Shear Stress

The bottom shear stress due to the combination of waves and currents can be calculated using the quadratic stress law (Christoffersen and Jonsson, 1985):

$$\tau = \rho_w \left(C_{f,c} u_c^2 + C_{f,w} u_w^2 \right)$$

Where

τ = bottom shear stress

ρ_w = density of water

$C_{f,c}$ = bottom friction coefficient for currents

u_c = maximum current velocity

$C_{f,w}$ = bottom friction coefficient for waves

u_w = maximum bottom velocity due to waves

An example is provided below using the results for the EFDC model grid cell along the western berm with the highest computed bed shear stresses due to currents as computed by the EFDC model. In the example, the maximum bed shear stress due to flows computed by the model are added to the computed bed shear stresses due to waves, and a stable particle size is determined based on those stresses. The stable particle size is computed for the 25-year and 100-year return-interval flow events conservatively assuming that the 100-year return-interval wave occurs at the same time as these events.

For the 25-year return-interval flow event, the computed bed shear stress is 6.33 Pascals (0.132 pounds per square foot) for the model grid cell. For the 100-year return-interval flow event, the computed bed shear stress is 14.2 Pascals (0.298 pounds per square foot) for the model grid cell.

The bottom friction coefficient for waves is computed using (van Rijn, 1993):

$$C_{f,w} = 0.045\left(\frac{u_w A_w}{\upsilon}\right)^{-0.2}$$

Where

$C_{f,w}$ = bottom friction coefficient for waves

u_w = maximum bottom velocity due to waves

A_w = peak orbital excursion

ν = kinematic viscosity of water

Maximum bottom velocities and peak orbital excursions for the 100-year return-interval wave were computed with water depths over the western berm set equivalent to the 25-year and 100-year return-interval flow events using the *Linear Wave Theory Module* in ACES. Based on this analysis, the estimated bed shear stress due to waves is 4.91 Pascals (0.103 pounds per square foot) for the 25-year event and 0.494 Pascals (0.0103 pounds per square foot) for the 100-year event. The shear stresses due to waves are higher for the 25-year return-interval flow event as compared with the 100-year return-interval flow event because the water depths over the berm are lower. Table 1 below summarizes the results of this analysis:

Table 1

Summary of Combined Forces from Currents and Waves

Flood Flow Return-Interval	Forces from Currents			Forces from Waves					Combined Forces	
	Maximum Depth-Averaged Velocity Computed by EFDC Model (m/s)	Maximum Shear Stress Computed by EFDC Model (Pa)	Maximum Shear Stress Computed by EFDC Model (psf)	Peak Orbital Velocity Computed in ACES (m/s)	Peak Orbital Excursion Computed in ACES (meters)	$C_{f,w}$	Computed Shear Stress For Waves (Pa)	Computed Shear Stress For Waves (psf)	Combined Shear Stress due to Waves and Currents (Pa)	Combined Shear Stress due to Waves and Currents (psf)
25-year	1.19	6.33	0.132	0.684	0.234	0.0105	4.91	0.102	11.2	0.235
100-year	2.12	14.2	0.298	0.163	0.0560	0.0186	0.494	0.0103	14.7	0.308

Notes:
m/s = meters per second
Pa = Pascals
psf = pounds per square foot

The stable median diameter (D_{50}) for particles subject to a given shear stress can be estimated based on the approach described by Shields (1936). The correlation between shear stress and particle size presented below represents the point at which the subject particle begins to move or "rock" on the bed and does not necessarily imply significant transport of particles of this size. In addition, Shield's work is based on a bed of uniform particles and does specifically account for the increased stability resulting from a well-graded armor layer constructed from a range of angular particles.

$$\tau_{*c} = \frac{\tau_c}{(\gamma_s - \gamma)D_{50}}$$

Where

τ_{*c}	= critical shear stress parameter (pounds per square foot)
τ_c	= critical shear stress (threshold of motion) (pounds per square foot)
γ_s	= specific weight of the particle [pounds per cubic foot (pcf)]
γ	= specific weight of the water
D_{50}	= median particle size (feet)

Shields provides a plot of dimensionless critical shear stress versus a dimensionless Reynolds number. This graphical representation, commonly known as the Shields diagram, is widely used to determine a general relationship for incipient motion. Rouse (1939) fitted a mean curve to the zone of these data points, above which particles are considered to be in motion, and showed that at higher values of the Reynolds number (i.e., coarse sediments/larger grain sizes, and/or fully turbulent flow), the critical shear stress parameter approaches a constant value of 0.060. Since then, others have proposed more conservative values for the critical shear stress parameter ranging from 0.039 by Laursen (1963) to 0.045 by Yalin and Karahan (1979).

Rearranging the equation above to solve for median particle size, and substituting a recycled concrete specific weight of 145 pcf (and assuming that the wave event occurs during freshwater flow event) and a conservative critical shear stress parameter of 0.039, yields the relationship below.

$$D_{50} = \frac{\tau}{3.2}$$

The maximum combined bed shear stresses for combined waves and currents for the 25-year and 100-year return-interval events are 0.235 pounds per square foot and 0.308 pounds per square foot, respectively. The median particle size (D_{50}) to resist the combined waves and currents ranges between 0.9 and 1.2 inches using this method, which is lower than the design median particle size of 6 inches that was selected to resist breaking waves.

Method 2 – Orbital Velocity Shear Stress

Another method to evaluate the stable particle size to resist the combination of currents from waves and flood flows is provided in Maynord (1998):

> "Significant wind wave activity can create large bottom velocities that can erode an unprotected sand cap. To define the required armor layer size to prevent scour, Equation 5 should be used with the maximum horizontal bottom velocity from the wave. For orbital velocities beneath waves, a C_3 = 1.7 is recommended."

Using Equation 5 from Maynord (1998) with C_3 = 1.7, as recommended, to represent the contribution from orbital velocities, the following equation can be used to compute D_{50} to resist currents from waves:

$$D_{50} = \frac{\left(\frac{V}{C_3}\right)^2}{g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)}$$

Where

V	= maximum horizontal bottom velocity from the wave
C_3	= 1.7 for orbital velocities beneath waves (page A-13 from Maynord 1998)
γ_s	= unit weight of recycled concrete
γ_w	= unit weight of freshwater
g	= 32.2 ft/s^2

Conservatively adding the maximum depth-averaged velocities predicted by the EFDC model to the maximum bottom orbital velocity for waves and substituting that value into the above equation, the computed D_{50} is 3.7 inches for the 25-year return-interval event and 5.5 inches for the 100-year return-interval event. These values are also lower than the required median grain size of 6 inches that was determined to resist breaking waves.

Both example calculations demonstrate that the selection of B/C armor material (with a D_{50} of 6 inches and a D_{100} of 12 inches) to withstand breaking waves will also more than adequately withstand combined currents from waves and flood flows.

Questions 5 and 6 Summary

As described in USACE (1994):

> "Equation 3-3 gives a rock size that should be increased to resist hydrodynamic and a variety of nonhydrodynamic-imposed forces and/or uncontrollable physical conditions. The size increase can best be accomplished by including the safety factor, which will be a value greater than unity. The minimum safety factor is $S_f = 1.1$."

As described in Appendix I of Anchor QEA (2010), the two-dimensional EFDC model was used to predict the local depth-averaged velocities and water depths spatially over the TCRA during several extreme events. While the EFDC model provides local velocities, the increase in the safety factor to a minimum of 1.3 was considered appropriate and conservative to account for these potential small-scale variations.

The TCRA cap also includes an Operations, Monitoring, and Maintenance (OMM) Plan to periodically inspect the site and address any issues that might arise from small-scale effects on the cap. This monitoring program currently includes quarterly visual inspection of exposed surfaces of the armored cap, combined with topographic and bathymetric surveys of the armored cap. A quantitative comparison of survey results is completed at each inspection to identify potential areas of cap thinning. If deficient areas of the cap are identified, the OMM Plan requires additional inspections, and expeditious development and implementation of corrective measures. Pre-tested stockpiles of armor rock C and armor rock D materials are stored at a nearby location to complete any maintenance activities. Because these two armor sizes are the largest of the four types of armor used in the cap, they can also be conservatively substituted for armor rock A and armor rock B/C for maintenance activities in any area of the cap. The same OMM activities are required if a 25 year storm or greater occurs between scheduled quarterly monitoring events.

We hope the above responses to your questions address any remaining concerns you may have on the TCRA design. Please let us know if you would like to discuss anything further.

Sincerely,

David C. Keith

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Anchor QEA, LLC

Cc:
Barbara Nann – United States Environmental Protection Agency
Philip Slowiak – International Paper Company
David Moreira – McGinnes Industrial Maintenance Corporation

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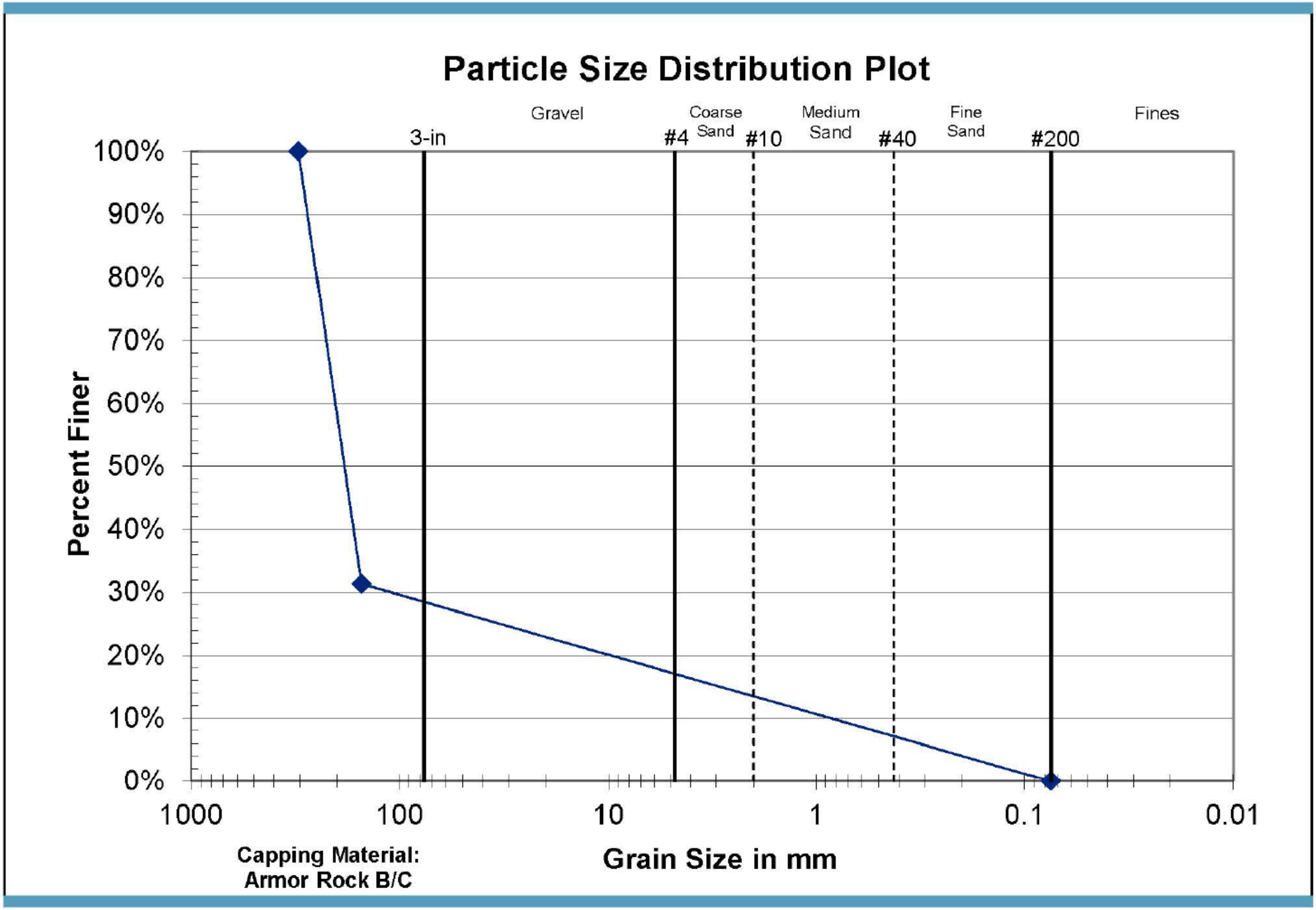

Figure 1
Gradation of Armor Rock B/C
San Jacinto River Waste Pits Time Critical Removal Action